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THE DELAWARE MEMORIAL BRIDGE: DESIGN PROBLEMS

By Charles H. Clarahan, Jr., and Elmer K. Timby,
Members, ASCE

CONSTRUCTION–STRUCTURAL DIVISIONS

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PAPERS

THE DELAWARE MEMORIAL BRIDGE: DESIGN PROBLEMS

By Charles H. Clarahan, Jr.,[1] and Elmer K. Timby,[2] Members, ASCE

Synopsis

The Delaware Memorial Bridge, at the southern end of the New Jersey Turnpike, is 10,756 ft long, including a major suspension span with girder and truss approaches. Specifications governing the design of the bridge elements are derived and discussed in this paper.

Subsurface conditions governing pier design are described, and settlement problems that required solution are introduced. The authors stress the need for extensive research in suspension bridge design to foster economy and safety in construction.

Preliminary Design

General Description.—The Delaware Memorial Bridge crosses the Delaware River between Deepwater, N. J., and a point in Delaware south of Wilmington. It supports two 24-ft roadways for a length of 10,756 ft. The sixth longest suspension bridge that has been built to date (1952) comprises about three eighths of that length. The approaches include ten deck truss spans 336.5 ft long and 3,460 ft of continuous deck girder spans, each 116.7 ft or 93.3 ft long. The two 24-ft roadways are separated by a 3-ft-wide median strip with a stepped curb, and 3-ft-wide safety walks are provided on either side, giving a width between railings of 57 ft on the approach spans. The safety walks on the suspended spans are 3.7 ft wide. The funds available for the project were limited. Although it is conservative in every respect, the design necessarily had to be economical as well.

Design Criteria.—The design of a conservative suspension bridge that is economical as well, today, must depend on the designer's judgment of what is conservative. He has no generally-accepted and well-defined basis of design to guide

Note.—Written comments are invited for publication; the last discussion should be submitted by February 1, 1953.

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him in the quantitative requirements for several important characteristics. Relatively few suspension bridges have been built with a span of more than a quarter of a mile and no one knows whether those bridges are overdesigned or underdesigned.

The behavior of the original Tacoma Narrows Bridge, in the State of Washington, and several bridges designed at the same time have indicated that plate girders are not desirable for the stiffening elements of long suspension bridges. The failure of the Tacoma Narrows Bridge demonstrated that too little was known as to how stiff a stiffening system should be. Wind load has shown itself to be a dynamic factor in design but there are no known means of translating the air movements at a given locality into the characteristics that a bridge must embody. Something is known about this and related problems but sufficient research is not available to establish new rules of design for large suspension bridges.

The net result is that adequate bridges can be designed, but there is not enough reliable information available to justify highly refined studies of ultimate economy. Economy is an important factor for the simple reason that a small percentage of a large construction cost is an appreciable sum of money; but, the first and most important factor is adequacy.

Most suspension bridge designers agree that a prime requisite for a conservative design is a double lateral system that, properly proportioned, will eliminate torsional deflections of the suspended structure of sufficient magnitude to harm the structure or to cause concern among the users of the bridge. In addition, the structure should be wide enough to prevent undesirable lateral deflections and should have sufficient vertical stiffness to prevent undesirable vertical deflections. There are no rigid standards to determine with precision what lateral or vertical deflections are undesirable, except by a study of the behavior of existing bridges.

It would be desirable for a bridge to have inherent aerodynamic stability. However, a double lateral system, properly proportioned and with a reasonable distance between the two planes of laterals, can provide torsional rigidity sufficient to prevent the resonance that would otherwise make a suspended span aerodynamically unsafe.

Early in the studies of the basic features of the suspended structure of the Delaware Memorial Bridge, the use of two planes of laterals was established as a necessary requirement. The required spacing of the stiffening trusses and cables to provide the desired width of roadway was 61 ft. The ratio of span to width is 35, which is about three quarters of the same ratio for the Golden Gate Bridge in San Francisco, Calif. There was no reason to be further concerned about the width of the bridge.

Considerable thought was given to the depth of the stiffening trusses. A minimum chord of about 70 sq-in. effective gross area was considered satisfactory for transverse wind loads. The shape of the chord section was largely determined by the width of the truss verticals. The connections of the suspenders to the verticals of the stiffening trusses required the use of a 21-in.-wide flange beam for the vertical. The problem then became one of determining the vertical spacing of the chords. A study of other suspension bridges seemed

to provide the only answer. Most of the long suspension bridges have only one plane of laterals and therefore their behavior in wind should be less satisfactory than with two planes of laterals. However, it was decided that this advantage would be largely discounted and kept as an additional factor of safety. A comparison of the vertical stiffness of numerous suspension bridges and their behavior in high winds indicated that this bridge, which has a weight of about 11,200 lb per ft, should have a moment of inertia of about 28,000 in.^{2}-ft^2 in two trusses. This rigidity is considered the minimum for a similar bridge with one plane of laterals, but it is believed to be amply conservative for a bridge with two planes of laterals. To obtain a moment of inertia of about 14,000 in.^{2}-ft^2 per truss with chord areas of 70 sq in. required a depth of 20 ft, which also provided satisfactory spacing between the two planes of laterals.

From architectural and economical considerations it was desirable to place the top chord of the stiffening truss above the sidewalk. This location permitted a reduction in the elevation of the roadway above the water, a decrease in length of the approaches, and a decrease in the width of the structure, since the top chord functions as part of the railing. The side elevation of the structure presents a cleaner appearance, eliminating the interference of a heavy railing with a functional truss. However, the top lateral system is necessarily eccentric to the top chord, which produced complications in design and erection of the bridge but not in its functioning.

Floor System.—With some of the main features of the suspended structure thus determined, it became possible to study the floor in greater detail. Several panel lengths were evaluated. Since the maximum spacing of suspenders was considered to be about 50 ft, the most economical panel length for the floor was found to be half that distance. The panel length finally adopted was a little less than 26 ft.

Considerable attention was directed to the material to be used in the floor. It is impossible to be certain what exact relationship exists between the weight of a given suspension bridge and the required rigidity of the stiffening trusses, but there is reason for believing that a relationship exists. Certainly a study of the behavior of existing suspension bridges in high winds seems to indicate such a relationship. Some attention was given to the use of a lightweight roadway slab, but a 7-in. reinforced concrete slab of conventional design was finally selected as providing the best surface. The increase in weight over that of a lightweight floor was considered to be of some benefit and the extra cost resulting from this weight was partly offset by the reduction in cost of the roadway slab as compared with lightweight floors. The median strip, with its stepped curb, would have been very heavy in concrete, and therefore was made of steel. The sidewalk, which is only an emergency and maintenance walk, is built of heavy grating to facilitate snow removal and reduce weight. The steel curbs are open.

It is customary to use expansion joints at frequent intervals in the floor system of suspension bridges to reduce the longitudinal bending in the floor beams because of changes in the length of the stiffening truss chords caused by changes in stress. Studies of these bending stresses indicated that a spacing

of 104 ft was satisfactory between these joints. Because of the frequency of the joints, the stringers are placed on top of the floor beam and designed for continuous action over four spans. Such a design eliminates the undesirable stresses produced in a continuous floor slab when the stringers frame into the floor beams as simple spans.

The floor beams were studied as trusses and as plate girders with brackets or sway bracing to the bottom lateral system. The trussed floor beams proved slightly more expensive but were adopted because they have less wind area, they have smaller deflections, and they provide very stiff cross frames for the torsion-resisting structure.

Logically, the top laterals were made the depth of the top chord of the floor beams. The bottom laterals and the bottom chord of the floor beam were made the depth of the bottom chord of the stiffening truss.

The preliminary selection of the depth and chord area of the stiffening trusses has been discussed. It was desired to use I-sections for the diagonals and the use of one and two diagonals per panel was studied. For reasons of economy, it was decided to use only one diagonal. The use of a single diagonal also somewhat reduces the apparent depth of the truss and facilitates maintenance.

Cable Design.—The cable sag ratio of the center span was made one tenth. An increase in sag would have produced a more flexible structure and a decrease in sag would have increased the size and cost of the anchorages. The normal sag ratio of one tenth is a desirable compromise.

With a tentative design of the stiffening trusses, the maximum chord stresses from live load were determined by the Hardesty-Wessman method of approximations[3] and the stresses resulting from wind on the center span were determined using the approximation[4] procedure introduced by E. L. Pavlo, M. ASCE. These approximate methods are quite satisfactory for preliminary designs, although it would be desirable to have the Hardesty-Wessman approximations extended to include the maximum shears. These approximations indicated that carbon steel would be satisfactory for the center-span trusses but that the greater part of the side spans had excessive unit stresses for carbon steel and slightly thicker silicon steel sections were required.

The towers are similar to those adopted for the Bronx-Whitestone Bridge, in New York, N. Y. The simplicity of their design is ideal for bridges of this size. It is possible that some slight saving could be obtained with fully-braced towers but the expenditure of a moderate additional sum to eliminate diagonal bracing seems fully warranted.

Final Design

Specifications.—The final design of the suspension bridge was based on the following design specifications (in general, which followed the determinations just described, referring to the 1944 official specifications of the American Association of State Highway Officials—AASHO):

[3] "Preliminary Design of Suspension Bridges," by Shortridge Hardesty and Harold E. Wessman, *Transactions*, ASCE, Vol. 104, 1939, p. 579.

[4] Discussion by E. L. Pavlo of "Suspension Bridges under the Action of Lateral Forces," by Leon S. Moisseiff and Frederick Lienhard, *ibid.*, Vol. 98, 1933, p. 1107.

Live Load.—For the approaches and the floor of main spans, H20-S16 loads were specified. Stiffening trusses, cables, towers, and anchorages were designed for a load of 2,250 lb per lin ft of bridge.

Exposed Areas for Determining Wind Loads.—The exposed area for application of transverse wind loads was considered to be the area of the structure as seen in side elevation plus the area of the leeward truss, railings, suspenders, cables and hand ropes, and leeward tower columns except the parts shielded by the floor system. The exposed area of all ropes and cables was taken as two thirds of the projected area based on the gross diameter.

The exposed area for the application of longitudinal wind loads was considered to be one half of the area of all truss members, floor beams, laterals, sway bracing, stringer diaphragms, suspenders, and railing posts as seen in cross sections of the suspended structure, and the entire area of towers as seen in elevation.

Wind Pressures.—The wind pressures on the exposed area were assumed as moving loads of the following intensities: (*a*) 30 lb per sq ft on the suspended spans and (*b*) 35 lb per sq ft on the towers, cables, and suspenders.

Wind Loads, Quartering Winds.—In considering quartering winds, the assumed simultaneous effect was based on seven tenths of the computed transverse wind load plus seven tenths of the computed longitudinal wind load.

Temperature Changes.—Two stipulations involving temperature changes were: (1) For stresses resulting from temperature changes, a drop in temperature of 58° F (from 68° F to 10° F) or a rise in temperature of 42° F (from 68° F to 110° F). (2) For movements caused by temperature, a drop of 78° F or a rise of 62° F.

Normal Unit Stresses.—Normal unit stresses were as required by AASHO.

Allowable Unit Stresses.—Normal unit stresses were specified for the floor system. For the stiffening trusses, allowable stresses of 1.1 times the combined normal unit stresses for dead load, settlement, temperature, and live load were used, as well as 1.25 times the normal unit stresses for dead load, settlement, temperature, and wind combined. For laterals allowable stresses of 1.25 times the combined normal unit stresses for dead load, settlement, temperature, and live load were used. The allowable stresses for the towers were the normal unit stresses for dead load, settlement, temperature, and live load combined, or 1.25 times the normal unit stresses for dead load, settlement, temperature, and wind combined. For bending in two directions, the allowable unit stresses in compression were increased in the lower quarter of the tower to vary uniformly from the allowable unit stress in compression of normal or 1.25 times the normal unit stress at a point one quarter of the height of the tower above the base plate to the allowable stress in tension of normal or 1.25 times normal unit stress at the top of the base plate.

Some specifications for suspension bridges have included combinations of live load with one half of the wind load—and full wind load with one-half live load at higher unit stresses, but in this case it was considered preferable to design for only a heavy live load or a high wind at moderate unit stresses.

Since there is a great reduction in highway live loads during storms, it is deemed unnecessary to combine live load with high winds.

Computations.—The detailed calculations were largely conventional. Live-load stresses in the stiffening trusses were computed by the deflection theory. The stresses in the side-span trusses were also computed, for loadings over the entire span, by simultaneous equations to determine the effect of having the anchorage saddle set back 32 ft from the truss support. Wind stresses were determined by simultaneous equations following the Moisseiff-Lienhard analysis.[5] Stresses caused by longitudinal deflections of the tower were found by successive approximations of the deflection. Transverse wind stresses in the towers were computed by a modification of the slope deflection method to introduce the effect of eccentricities caused by deflections, in a manner similar to the method of anysis developed for the Golden Gate Bridge towers.

Foundation Conditions

Subsoil Characteristics.—At this location, rock is several hundred feet below the surface. The upper portion of the ground consists of deposited strata of sand, gravel, and mud to varying depths of less than 100 ft. Below these deposits are the firmly compacted beds of clay, sand, and gravel of the Cretaceous, Tertiary, and Quarternary ages, with nearly horizontal stratifications, previously compacted by much higher loadings than exist at the time construction was started. For estimating the bearing values of these lower soils, the angle of internal friction was assumed to be 32°, which is well below the average values obtained from direct shear tests.

Foundation Details.—The east anchorage is founded at about El.-70 on a thin layer of compact varved sand, clay, and silt over compact gray sand. The west anchorage, founded at El.-96, is on a thick bed of varved clay, silty sand, and lignite except for the southeast corner, which is carried on medium to fine gray sand. The site was crossed by an erosion gulley running diagonally across the foundation. The depth of the gulley extended to approximately El.-87 nearly 50 ft outside the anchorage. The west tower pier landed on hard red and gray clay at El.-87 and the east tower pier on similar material at El.-115.

In general, foundation pressures are moderate, giving factors of safety of at least two, except temporarily under the rear of the anchorages during erection conditions and under the tower piers during wind loadings. In these two cases, the factor of safety was reduced to 1.8.

Settlement.—Naturally, with weight concentrations of the magnitude of these foundations, rather large settlements could be expected, as the loads imposed on the materials described changed during erection and will continue to change until conditions of equilibrium are re-established. The computed settlements reach ultimate values as high as 1 ft. The design provided that the computed increments of settlements after cable spinning began were to be quite closely equal for all the piers. To compensate for the settlements, a band of concrete 8 in. high was added at the base of the upper part of the anchorage and

[5] "Suspension Bridges under the Action of Lateral Forces," by Leon S. Moisseiff and Frederick Lienhard, *Transactions*, ASCE, Vol. 98, 1933, p. 1080.

the tower below the roadway level was increased 8 in. in height. The calculated settlements showed a probable rotation of the anchorage saddle toward the towers, which would produce a deflection of the towers toward the channel. Because of these settlements, the design longitudinal deflections of the towers were increased by 0.25 ft in either direction as the probable range of field errors in securing design dimensions and by an additional deflection of 0.2 ft toward the channel to allow for the effects of computed settlements.

The anchorages tilted toward the shore during their construction, as predicted because of heavier soil pressures under the rear of the anchorages. To obtain the design dimensions, the anchorages were built to the plane of the top of the anchorage caissons instead of to a horizontal plane. From the computed settlements, it was predicted that the anchorage saddles would be located 2 in. farther from the towers than the design dimension at the time of starting cable spinning. Measurement of this distance at the time cable spinning began confirmed this prediction closely.

If the calculated settlements are accurate, the bridge will be at its design elevation about two years after it is completed and will ultimately be about 2 in. low. During the last year of construction, however, the increments of settlement were about 50% of the calculated settlements and the bridge is now expected to remain slightly higher than design elevations. Because of the unusual foundation conditions for a bridge of this magnitude, the settlements are recorded every month and are studied carefully. To date (1952), there has been no reason to suspect that the design of the piers is not completely satisfactory and conservative in all respects.

Design Details.—The tower piers are simple in design. The caisson is larger than the pier top because of soil conditions, giving the cross section the appearance of a milk bottle.

The anchorage was developed gradually through a series of studies to develop an economical, conservative design. The first studies were made with the eyebars arranged in seven vertical rows, which gave a wide anchor block. In the development process it became apparent that a narrower anchorage block, with the eyebars arranged in five vertical rows, was better adapted to the final shape of the anchorage. In general, the top part of the anchorage at the rear consists of two anchor blocks of concrete surrounding the two anchor chains, a heavy wall connecting the blocks, and two masses of concrete over the anchor blocks. The resultant of the weight of this concrete and the load on the anchor chain is a horizontal load on two large girders in the vertical plane of the cables that extend to the high columns at the front of the anchorage. These columns support both the saddles at the turning point of the cables and the reactions for the side spans. The front faces of these columns are sloped, in order to keep the resultant loads well inside the columns. The upper part of the anchorage, as described previously, rests on a 10-ft slab of concrete that is heavily reinforced. The lower part developed gradually during design, changing from two or three caissons into a single unit believed to have the largest area of any caisson in use up to that time. Bidding plans were developed for both two caissons and one caisson. The single caisson proved slightly cheaper.

Approaches

Numerous studies were made to develop an economical design of pleasing appearance for the approaches. These studies were conventional and similar to the studies made for any long bridge. Some minor possible savings were rejected to eliminate a multiplicity of span lengths and depths of structure.

Design specifications were practically the same as those of the AASHO (1944) with the exception of the omission of the combinations of live load and 30-lb wind, which seemed unjustified for girder and truss bridges carrying four lanes of traffic. Ordinarily the combination does not govern the design in such cases and only increases the time required to make a design.

The median strip and safety walks were designed to carry the standard truck wheels at 50% over the normal allowable unit stresses. The curb was designed for a lateral load of 1,000 lb per lin ft, or a concentrated lateral load of 6,000 lb. The lower part of the railing was designed to carry a concentrated lateral load of 10,000 lb applied to 2 ft above the sidewalk.

These loads on the sidewalk, median strip, curb, and railing are much higher than the AASHO specifications but were believed to be essential for a bridge of this size.

Model Tests

As usual, studies during the design of this bridge re-emphasized the need for more exact design criteria. Properly proved theoretical methods would make it possible to eliminate that part of the safety factor required by lack of specific information on structural requirements. With the intention of adding slightly to the knowledge in this field, a research program was sponsored by the designers at Princeton University, at Princeton, N. J., to study the problem of torsional rigidity and stress distribution in the suspended structure of a suspension bridge. The final design of the Delaware Memorial Bridge was used as the prototype and a nearly exact model of twelve panels at one-twelfth scale was built. The members were built up with continuous and spot welds and the connections were made with machine screws and nuts tightened with a torque wrench.

Although the tests and theoretical calculations are planned to extend over several years, tests have already been made with a single lateral system (the top lateral system of the prototype) and with a double lateral system. The tests were made by applying a static torque at each end of the model with all forces vertical. The torsional deflection with a double lateral system was found to be only 5% of the deflection with a single lateral system. For this loading the structure is statically determinate to the same degree as a simple-span truss in which secondary stresses are ignored. The stresses in all laterals are equal and can be found by equating the forces on a single-vertical truss. The truss is subjected to a torque (in a longitudinal direction) composed of two equal and opposite vertical forces, one at each end. This torque is resisted by an equal and opposite torque composed of horizontal forces having an arm equal to the distance between centers of the laterals. The horizontal force is the sum of the components of one plane of the laterals parallel to the truss. As previously mentioned, the stresses in all laterals are equal. The stresses

in the vertical truss are those caused by the applied forces at the ends of the truss and the components of the laterals. The test data of the model closely checked the theoretical calculations of the stresses and deflections.

Much more work is necessary to develop a simple method of computing the torsional stresses in a suspension bridge. In these model tests the corners of the model were free to deflect vertically and horizontally. In a suspension bridge there are restraints that always prevent vertical motion and sometimes horizontal motion at one end of a span. In addition, the cable also acts to produce restraints at the suspender points. The effect of these restraints must be determined.

The tests on the model have confirmed the belief that a second plane of laterals greatly increases the torsional strength of the suspended structure. Much work remains to be done on this one phase of the design problem. Other phases require similar detailed attention. The writers are of the opinion that: (1) The development and confirmation of the necessary criteria can only be accomplished through a well-planned program of research extending over several years; (2) such development and confirmation are economically sound subjects for research; (3) much more needs to be known about wind load characteristics at particular sites; and (4) large-scale models of suspended structures, towers, and topography should form a part of the program.

Conclusion

In summary, it can be stated that the Delaware Memorial Bridge is designed conservatively and is stable and adequate to resist all probable loads. If additional reliable criteria had been available to the designers, it might have been possible to achieve some slight economies with assurance that safety would not thereby be impaired. Until such time as well-established criteria are available, designs based on past satsifactory performance are mandatory for major suspension bridges.

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